6. SLOPE (LAND STABILITY) ASSESSMENT

6.1 Introduction

This section addresses outstanding issues raised by the screening process regarding land stability (see Table 1).

The basement is to be constructed using underpinning techniques constructed in a 1,4,2,5,3 sequence in bays of maximum 1.2m length. The underpins will be constructed in a single lift with a toe projection at the base, forming an L-shaped gravity retaining wall in the temporary condition to resist sliding, overturning and excessive bearing pressures. The underpins will be constructed in trenches with a central soil mass retained to provide support for temporary props and formwork. The underpins will be supported in the permanent condition by the basement slab.

Structural details and typical construction sequences are provided in Appendix B.

6.2 Adjacent Structures

The basement shares a party wall with No. 2 Hampstead Square. From available photographs it appears this property has a single storey basement, with a lightwell evident along the frontage to Hampstead Square. For the purposes of this assessment it is assumed the basement does not extend to the party wall between No. 2 and No. 4. This is a reasonable worst case as the ground movement calculations assume the party wall underpin will be retaining soil to ground level.

No. 5 Hampstead Square to the south of the site is not a party wall but its foundations are located in close proximity (circa 0.5m to 1m away). Again it is not confirmed this has a basement, and has been assumed not to in the calculations. The RHH calculations assumed a surcharge for the foundation of 60kPa, and this has been maintained for consistency.

The extent of adjacent basements should be confirmed.

6.3 Ground movements arising from basement excavation

During excavation the soils at formation level will be subject to stress relief as some 3.5m to 4m of overburden are removed. Due to the granular nature of the soils, they are likely predominantly non-plastic and therefore not subject to significant volume change over time on unloading. Given that the soils are predicted to behave as drained materials, any minor heave movements in the form of elastic recovery that may occur will be removed during levelling for casting of the basement slab. No long term heave is predicted.

6.4 Underpin Walls – Global Stability

6.4.1 General

R.H.Horwitz Associates (RHH) have completed gravity wall designs at all relevant underpin sections to ensure the walls are stable in sliding, overturning and ground bearing in both the permanent and temporary condition. It should be noted the calculations were completed assuming a traditional lumped factor of safety method as opposed to the partial safety factor method in accordance with EC7, as below:

- Factor of Safety for overturning = 2.0
- Factor of Safety for sliding = 1.5
- Maximum allowable pressure < Allowable Bearing Pressure (200kPa)

A number of assumptions were made, particularly with regard to design parameters as no site investigation data was available at the time. For the Bagshot Sands they assumed a friction angle of 34°, unit weight of 20kN/m^3 and an allowable bearing pressure of up to 200kPa. These values are considered appropriate for design (see Section 4.5). Groundwater was taken at 2.35mbgl or 1m above the base of the wall toe.

The results revealed the walls are stable in the permanent condition, but prone to overturning and sliding failure in the temporary condition primarily due to the line loads from the foundation not being applied, and thus not providing adequate base friction and / or restoring moment. Temporary prop loads and levels were calculated by RHH based on the results of the retaining wall design to resist these failure modes.

6.4.2 Analysis

A design check using Geosolve GWALL analysis software was used to confirm the design and ensure global stability in the temporary condition at two critical sections: the party wall (Underpin D) and along the southern wall section (Underpin E). It is assumed that the wall ground interface coefficient is 0.67 for both base and wall friction. The walls have been modelled at the top of the underpin, rather than at ground level as in the RHH calculations, with an additional surcharge from the soils above the top of the wall.

GWALL has the capability to model horizontal loads as 'anchor' loads. The calculated prop loads by RHH have been analysed as anchor loads to mimic the effect of the propping.

The results are summarised below in Table 9 :

Table 9: GWALL Analysis Summary

Section	**RHH Prop load @ 1.5m (kN/m)**	**GWALL FoS against sliding**	**GWALL FoS against overturning**
Wall F – Underpin D (party wall to No.2)	50	1.78	3.07
Wall G – Underpin E (adjacent to No.5)	110	3.05	4.87

The results indicate with the additional prop loads the walls are stable against sliding and overturning in the temporary condition. Additionally, given that a soil mass will remain in the centre of the basement excavation, with underpins excavated in shored bays, there will be additional resistance to sliding that will ensure short term stability.

Full GWALL output is provided in Appendix G.

6.5 Underpin Walls – Lateral Movements

6.5.1 General

Two critical sections have been analysed in areas where adjacent properties may be affected to determine lateral wall deflections in the permanent condition. As GWALL cannot calculate deflections, the underpin walls have been modelled as 300mm thick concrete diaphragm walls in Geosolve WALLAP embedded retaining wall analysis software. This software has the capability of calculating lateral deflections. Although the software is designed to analyse embedded walls, the underpin deflections can be reasonably modelled as a cantilever beam by assuming an 'embedment' within a high strength, stiff stratum to mimic the reinforced L-section between the wall and basement slab.

Given the lack of monitoring data, groundwater has been modelled at the ground surface as a worst case in the long term condition.

6.5.2 Analysis

The analysis revealed that horizontal ground movements are likely to be of the order of <1mm, generating negligible foundation settlements in the sensitive areas.

The amount of ground movement will depend largely on the quality of the underpinning workmanship. The WALLAP analysis has assumed a 'continuous' unreinforced mass concrete retaining wall installed instantaneously. The detailing and construction of the reinforcement and connections between underpin sections and basement slab will be important in controlling deflections.

Full WALLAP output is provided in Appendix H.

6.6 Underpin Walls – Vertical Movements

6.6.1 Soil Bearing Capacity

Theoretically an allowable bearing capacity of 300kPa is possible for the underpin bases in the permanent condition, taking into account the depth to groundwater (assumed to be at formation level), depth of overburden and lateral restraint provided by the basement slab, both of which will help to prevent shear planes developing beneath the underpin. Forster [9] recommends 175kPa for deep foundations in Barton Formation soils.

The line loads from the walls are to be transferred to the soils at depth via the underpin bases. Ground bearing pressures have been calculated by R.H.Horwitz Associates taking into account eccentric loadings on the underpin base derived from calculated moments about the base of the wall. A maximum bearing pressure of 155kPa was calculated at the party wall underpin, with all cases falling below the allowable bearing pressure of the soil.

6.6.2 Settlement

Settlements from the construction are likely to be negligible given the net loadings on the soil at formation level are minimal. Maximum unfactored ground bearing pressures from the R.H.Horwitz design calculations are summarised below in Table 10 , alongside assumed net bearing pressures with the associated stress relief from the excavation. Again it is assumed the unloading will be 75kPa across the basement footprint.

[9] Forster,A. *The Engineering Geology of the London Area TR WN/97/27* British Geological Survey August 1997

Table 10: Net Bearing Pressures and anticipated settlement

Wall Ref.	Underpin Ref.	Unfactored Wall Line Load (kN/m)	Underpin base length (m)	Max Ground bearing pressure (kPa)	Net bearing pressure (kPa)	Anticipated settlement (mm)
A	A	50.1	1.60	59	-16	Negligible
B	B	46.5	1.70	52	-23	Negligible
C*	C	79.4	1.00	79	4	Negligible
D	B	46.5	1.70	52	-23	Negligible
F	D	111.4	1.65	155	80	3 - 5
G	E	81.2	1.85	78	3	Negligible
H	F	81.2	1.50	102	27	1 - 2
J	G	62.6	1.50	62	-13	Negligible
Lightwells**		0	3.15	23	-52	Negligible

**Wall C is an internal wall and not retaining soils. Max. bearing pressure calculated assuming line load fully distributed through 1m wide underpin base.*

*** Lightwell bases are to be cast against already cast underpin bases to resist sliding*

NOTE : Wall references E, K and L are not being underpinned.

It can be seen that net bearing pressures are minimal and often lower than the existing vertical stress within the soils at formation level. The additional loads that will be transferred will therefore not generate additional settlement above that which will have occurred historically in the soil under the weight of existing overburden. As such estimated settlements are negligible and even in the worst case do not exceed 5mm (Wall F; Underpin D)

On the basis of the above damage associated with excessive settlements of the underpins is not considered to present a risk to adjacent structures.

6.7 Damage Category Assessment

Ground movements have been analysed based on the construction scheme as currently envisaged to provide indication as to the potential damage that may be caused to neighbouring structures and infrastructure.

The calculated ground movements have been used to assess potential 'damage categories' to the neighbouring properties. The methodology proposed by Burland and Wroth[10] and later supplemented by the work of Boscardin and Cording[11] has been used, as described in *CIRIA Special Publication 200*[12] and *CIRIA C580*.

Assumed damage categories are summarised in Table 11 below :

Table 11: Classification of damage visible to walls (reproduction of Table 2.5, CIRIA C580)

Category	**Description**
0 (Negligible)	Negligible – hairline cracks
1 (Very slight)	Fine cracks that can easily be treated during normal decoration (crack width <1mm)
2 (Slight)	Cracks easily filled, redecoration probably required. Some repointing may be required externally (crack width <5mm).
3 (Moderate)	The cracks require some opening up and can be patched by a mason. Recurrent cracks can be masked by suitable linings. Repointing of external brickwork and possibly a small amount of brickwork to be replaced (crack width 5 to 15mm or a number of cracks > 3mm).
4 (Severe)	Extensive repair work involving breaking-out and replacing sections of walls, especially over doors and windows (crack width 15mm to 25mm but also depends on number of cracks).
5 (Very Severe)	This requires a major repair involving partial or complete re-building (crack width usually >25mm but depends on number of cracks).

Maximum vertical settlements at the party wall are not anticipated to exceed 5mm. Assuming good quality workmanship, lateral movements have been calculated to not exceed 1mm given the relative stiffness of the underpin walls and reinforced connections to the underpin bases. This would indicate that damage categories to the adjacent

[10] Burland, J.B., and Wroth, C.P. (1974). *Settlement of buildings and associated damage*, State of the art review. Conf on Settlement of Structures, Cambridge, Pentech Press, London, pp611-654

[11] Boscardin, M.D., and Cording, E.G., (1989). *Building response to excavation induced settlement*. J Geotech Eng, ASCE, 115 (1); pp 1-21.

[12] Burland, Standing J.R., and Jardine F.M. (eds) (2001), *Building response to tunnelling, case studies from construction of the Jubilee Line Extension London*, CIRIA Special Publication 200.

structures are likely to be no worse than ‘Category 1’ comprising fine cracks that can be easily treated during normal decoration.

7. SURFACE FLOW AND FLOODING

It is noted in Section 5 of this report that there is no real material change to the ground water conditions on site, with the basement footprint formed within the existing external walls. There will be no additional surface water recharge or deviation of surface water flow routes as a result.

As already identified the site lies outside any EA designated Flood Zone and is not highlighted as a street that flooded in the 1975 and 2002 events.

Surface waters will join the existing drainage infrastructure (albeit via basement pumping if a gravity fed solution is not feasible), with no significant changes in drainage outflows anticipated from the site.

As such the development will have a negligible impact on surface water flow and flooding. In addition the basement is likely to provide enhanced attenuation given its requirement to be drained in accordance with building regulations.

8. CONCLUSIONS

The findings of this Basement Impact Assessment are informed by site investigation data already available for the site and structural drawings and calculations provided by Cranbrook Basements and RHH. On the basis of this information it is considered that the proposed development will not have a detrimental effect on groundwater or surface flooding in the vicinity of the site.

There is a potential for running sands where excavations are taken below basement formation level. This could potentially cause void formation and excessive settlement under existing and party wall foundations. A robust and effective contingency plan will need to be implemented should groundwater or running sand conditions be encountered.

The construction of the basement will generate ground movements due to a variety of causes, however, based on a numerical assessment it is considered that these movements can be controlled through appropriate construction techniques and control measures to limit building damage categories to no worse than ‘Category 1’ (very slight).

FIGURES